COST-EFFECTIVENESS OF STRONGER WOODFRAME BUILDINGS

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SUMMARY

A performance-based earthquake engineering methodology has recently been developed that quantifies building performance in terms of repair costs, life-safety risk, and loss of use ("dollars, deaths, and downtime"). The methodology is used to quantify the economic benefit (avoided future repair costs) of various detailed seismic retrofits, above-code design alternatives, and construction quality levels for several particular, completely designed woodframe buildings. Benefits are quantified assuming each house is located in any of California's 1,653 ZIP Codes. It is found that one example retrofit (costing approximately $1,400) exhibits benefit-cost ratios as high as 7.8, saving the homeowner up to $11,000 in avoided losses if the house were located in the highest-hazard area of the state. Four retrofit or redesign measures are cost effective in at least some locations. Higher quality is estimated to save thousands of dollars per house. We conclude that such quantitative benefit data could inform homeowners' decisions about mitigating seismic risk.

INTRODUCTION

Purpose of this paper. California's recent earthquake history, and particularly the 1994 Northridge earthquake, show that moderate earthquakes can be costly and deadly, and that losses in woodframe construction contribute substantially to both economic and life-safety risk. To mitigate this risk, it is worthwhile to examine the structural behavior and economic seismic performance of woodframe construction. The CUREE-Caltech Woodframe Project, funded by a $5.2M grant from FEMA, involved laboratory, desktop, and field studies of the structural and economic performance of residential woodframe construction (see [CUREE 2003] for an overview). The project entailed approximately 30 awards in five general thrust areas: laboratory testing, field investigations, building codes, economic modeling, and education. This paper summarizes the economic-modeling study, which assessed the cost-effectiveness of various levels of seismic design and construction quality, both new and retrofit, for woodframe residential buildings in California [Porter et al. 2002]. This paper also includes material from a subsequent study of benefit on a broad geographic basis. The research had three objectives: (1) A fundamental improvement in loss modeling. (2) Quantification of benefit of retrofit, above-code design, and construction quality. (3) Vulnerability functions and HAZUS-compatible fragility functions. (HAZUS fragilities are not treated here.)

USE OF VULNERABILITY FUNCTIONS IN SEISMIC RISK ANALYSIS

Seismic vulnerability, as used here, refers to a probabilistic relationship between the uncertain repair cost of a particular facility and the shaking intensity to which it is subjected in a single event. The economic benefit of a seismic retrofit or redesign measure can be measured in terms of reduction in future earthquake losses. It is denoted here by B, and calculated here as the present value of the difference between the expected annualized repair cost before and after retrofit or redesign, using Equation 1:

$$B = \left(EAL - EAL_r\right)\left(\frac{1-\mathrm{e}^{-\rho t}}{\rho}\right) = \left(V\int_0^\infty y(s)\left|G'(s)\right|ds - V_r\int_0^\infty y_r(s)\left|G_r'(s)\right|ds\right)\left(\frac{1-\mathrm{e}^{-\rho t}}{\rho}\right) \tag{1}$$

where *EAL* denotes the expected annualized repair cost, ρ and *t* are the real discount rate and planning period, respectively, *V* refers to the replacement cost of the facility, $y(s)$ refers to the mean seismic vulnerability function (repair cost divided by replacement cost as a function of shaking intensity *s*), $G(s)$ is the hazard function (mean frequency of exceeding *s*), and $G'(s)$ refers to its first derivative with respect to *s*. The subscript *r* indicates values after a retrofit or redesign. Since hazard can depend on period, $G_r(s)$ may differ from $G(s)$. In most practical circumstances, $y(s)$ and $G(s)$ are available at discrete values of *s*, and Equation 1 is evaluated numerically.

In some practical problems, it may be desirable to calculate *EAL* and *B* for a location where site soil classification is uncertain (e.g., for a site whose location is inexactly known, such as by ZIP Code), in which case *EAL* of Equation 1 can be calculated by

$$EAL = \sum_{sc=1}^{Nsc} p_{sc} V \int_0^\infty y(s)\left|G_{sc}'(s)\right|ds \tag{2}$$

where N_{sc} refers to the number of possible site classifications, p_{sc} denotes the probability of site classification *sc*, and the subscript *sc* on G' indicates that it reflects the hazard for site class *sc*. The value of EAL_r is calculated similarly, using *V*, *y*, and G' appropriate to the changed conditions.

Most of the parameters of Equations 1 and 2 are relatively easy to acquire. Hazard information can be acquired from US Geological Survey [Frankel and Leyendecker 2001]. Soil data are readily available for many states [e.g., Wills et al. 2000]. The values *V* and V_r can be estimated by standard construction cost-estimation techniques and approximated for many classes of facilities using publications such as [RS Means 2001a]. The challenge lies in estimating the seismic vulnerability functions.

Three general approaches have been used to estimate $y(s)$: empirical, analytical, and expert opinion, each with advantages and disadvantages. Empirical seismic vulnerability functions (created by regression analysis of historic loss data) are highly defensible, but are often proprietary, of poor quality, lack adequate information about interesting details, and are meaningful only for classes of buildings, not particular ones. Examples include [U.S. Coast and Geodetic Survey 1969], [Scholl et al. 1982], and [ATC 2001]. Seismic vulnerability functions based on expert opinion overcome difficulties in acquiring necessary historic loss data, but lack the defensibility of empirical data, an important issue for insurers and regulators. Examples include [Freeman 1932] and [ATC 1985]. Analytical techniques allow one to calculate the effects of interesting details for particular buildings, and are defensible on a theoretical basis, but can be difficult to validate. Examples include [Czarnecki 1973], [Kustu et al. 1982], and the assembly-based vulnerability (ABV) technique [Porter et al. 2001].

METHODOLOGY

ABV is employed here to calculate seismic vulnerability functions. The methodology meets the two main criteria set out by [Hamburger and Moehle 2000] for a second-generation performance-based earthquake engineering (PBEE) methodology, namely, system-level performance evaluation (e.g., economic losses or repair duration) and rigorous propagation of all important sources of uncertainty. Furthermore, it avoids structural-analysis simplifications required for pushover-type analysis. It employs only experimental information, state-of-the-art structural- and damage-analysis principles, and well-established construction cost-estimation procedures. It does not rely on expert opinion or other difficult-to-verify methods. The study reported in [Porter et al. 2002] is its first application to woodframe buildings and to the estimation of the economic benefit of a common seismic retrofit measure for houses.

ABV has been presented in detail in [Beck et al. 1999], [Porter et al. 2001], and elsewhere. In summary, the methodology has five stages: facility definition, hazard analysis, structural analysis, damage analysis, and loss analysis, as illustrated in Figure 1.

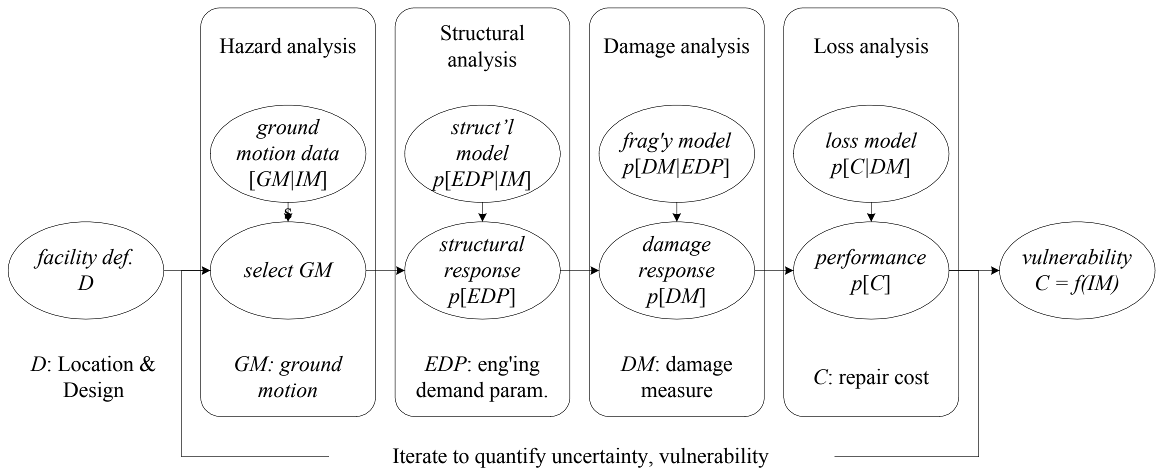

Figure 1 The assembly-based vulnerability (ABV) methodology, in schematic form

Facility definition. To define the facility, one must know its location (latitude and longitude) and design, including site soils, substructure, structural and nonstructural assemblies. An assembly is a collection of basic building components, assembled and in place, defined according to a standard taxonomic system. One creates an inventory of the damageable assemblies and identifies the engineering demand parameter (*EDP* such as interstory drift ratio, member force, etc.) that is the primary cause of damage to each assembly.

Hazard analysis (ground-motion selection). Shaking intensity is parameterized with an intensity measure, denoted here by *IM*, such as damped elastic spectral acceleration, S_a. The parameter s in Equations 1 and 2 would be a particular value of *IM*. For a given s, one selects a ground-motion time history and scales all of its accelerations by a constant to achieve $IM = s$. We use spectral acceleration at the facility's small-amplitude fundamental period of vibration, denoted by $S_a(T_1)$, as the *IM*, and limit scaling of recorded ground-motion time histories to a factor of 2.0 to achieve the desired *IM* level.

Structural analysis. One creates a structural model and, for each ground-motion time history, performs a nonlinear time-history structural analysis to determine structural response, quantified via *EDP*s. The structural model is stochastic in that masses, damping, and force-deformation behavior are uncertain, having prescribed probability distributions. In the present study, for example, we take the mass M and viscous damping ratio Z as normally distributed:

$$F_M(m) \equiv P[M \le m] = \Phi\left(\frac{m-\mu_M}{\sigma_M}\right), \qquad F_Z(z) \equiv P[Z \le z] = \Phi\left(\frac{z-\mu_Z}{\sigma_Z}\right) \tag{3}$$

where P denotes probability, Φ denotes the cumulative standard normal distribution, and μ and σ denote the mean and standard deviation of the variable in question.

Damage analysis. In the damage analysis, one simulates damage to each damageable assembly via one or more fragility functions. Each fragility function gives the probability that the assembly will reach or exceed a specified damage measure (denoted by *DM*) when subjected to a given level of *EDP*. *DMs* are defined in terms of the repairs required to restore the assembly to the undamaged state. The fragility function for a particular value of the damage measure is denoted by $F_{dm}(x)$. The cumulative distribution function of *DM*, given that $EDP = x$, is given by:

$$\begin{aligned} F_{DM|EDP}(dm \mid x) &\equiv P[DM \le dm \mid EDP = x] \\ &= 1 - F_{dm+1}(x) \qquad && dm \in \{1, 2, \ldots N_{DM} - 1\} \\ &= 1 && dm = N_{DM} \end{aligned} \tag{4}$$

where N_{DM} refers to the number of possible damage states (in addition to the undamaged state) of the assembly in question. See [Porter et al. 2002] for details of the laboratory data and fragility functions used in the present study. We take all fragility functions $F_{dm}(x)$ as lognormal cumulative distribution functions (Equation 5), with parameters x_m and β varying by assembly type and damage state.

$$F_{dm}\left(x\right) \equiv P\left[DM \geq dm \mid EDP = x\right] = \Phi\left(\frac{\ln\left(x/x_m\right)}{\beta}\right) \tag{5}$$

Loss analysis. Given damage, one calculates the damage factor y as

$$y = \frac{1}{V}\left(1 + C_{OP}\right)\left(\sum_{j}\sum_{dm} N_{j,dm} C_{j,dm}\right) C_L C_I \tag{6}$$

where C_{OP} refers to the (uncertain) contractor's overhead-and-profit factor; $N_{j,dm}$ refers to the number of assemblies of type j in damage state dm (determined in the damage analysis); $C_{j,dm}$ refers to the uncertain cost to restore one assembly of type j from damage state dm; C_L refers to the location cost factor (local construction costs as a factor of those in the location for which the $C_{j,dm}$ are calculated); and C_I refers to the inflation cost factor (construction costs in the year of interest as a factor of those in the year for which the $C_{j,dm}$ are calculated). Tabulated values of C_L and C_I are commonly available [e.g., RS Means 2001b]. We treat C_{OP} as uniformly distributed between a and b, and $C_{j,dm}$ as lognormal with mean and standard deviation varying by assembly type and damage state, i.e.,

$$F_{Cj,dm}\left(c\right) \equiv P\left[C_{j,dm} \leq c\right] = \Phi\left(\frac{\ln\left(c/x_m\right)}{\beta}\right) \qquad \begin{aligned} F_{COP}\left(c_{op}\right) &\equiv P\left[C_{OP} \leq c_{op}\right] \\ &= \left(c_{op} - a\right)/\left(b - a\right) \qquad a \leq c_{op} \leq b \\ &= 1 \qquad\qquad\qquad\qquad b < c_{op} \end{aligned} \tag{7}$$

where x_m and β vary by assembly type and damage state.

Propagation of uncertainty. We have examined several ways to calculate mean, variance, and higher moments of loss: Monte Carlo simulation (MCS), Latin Hypercube simulation, quadrature, and first-order, second-moment analysis. Because of an important limitation on current knowledge of the effects of the ground-motion time history on *EDP*, all four methods require MCS during the structural analysis. Structural analysis is the only computationally expensive part of the analysis. The study described here employs MCS throughout, so that each of the foregoing steps (after facility definition) is repeated many times, each time sampling each uncertain variable once. That is, in each analytical step, the cumulative distribution functions of Equations 3, 4, and 7 are inverted and evaluated at independent and identically distributed samples of U, where U is a uniformly distributed random variable bounded by 0 and 1. For example, in the present study, in a given simulation, the damping ratio Z is simulated using Equation 8:

$$z = \Phi^{-1}\left(u\right) \cdot \sigma_Z + \mu_Z \tag{8}$$

where Φ^{-1} is the inverse standard normal distribution and u is equally likely to be any number between 0 and 1. Repair cost is then evaluated using Equation 6. This process is repeated many times at each *IM* level of interest (in the present study, 20 times at each of 20 S_a values).

APPLICATION

Index Buildings and Variants

Index buildings We applied ABV to four hypothetical California woodframe building designs, each with four or more versions reflecting different quality of construction, retrofit, or redesign, for a total of 19 buildings, to which we refer as variants. The designs, shown in Figure 2, are referred to as the small house (Figure 2a), large house (Figure 2b), townhouse (Figure 2c), and apartment building (Figure 2d). These hypothetical but fully designed buildings were created by a panel of architects and engineers [Reitherman and Cobeen 2003]. See [Porter et al 2002] for details of the index buildings.

In summary, the small house is a single-story, 1,200-sf, single-family dwelling, with raised foundation and construction details appropriate to 1950s California housing. Its walls are stucco-finished exterior, gypsum-wallboard interior finish, and no structural sheathing. The large house is a single-family dwelling constructed around 1990. It is a 2,420-sf, two-story home with slab-on-grade foundation. Walls have stucco exterior finish, gypsum wallboard interior, and plywood or oriented strandboard (OSB) exterior structural sheathing. The townhouse is a hypothetical building constructed in early 1990s. Each of three units is a 2,000-sf, two-story dwelling with a slab-on-grade foundation. Exterior walls have stucco finish, many over plywood or OSB sheathing. Interior walls are finished with gypsum wallboard. The apartment building a hypothetical, 10-unit, 3-story dwelling constructed during the 1960s with ten 850-sf units. It has two levels of residential space above ground-level tuck-under parking. The foundation is slab on grade. Walls have stucco exterior finish, gypsum wallboard interior. Many walls have plywood structural sheathing. The longitudinal front wall is open on the ground level to provide access to parking spaces, producing a soft-story effect that has proven hazardous in recent earthquakes.

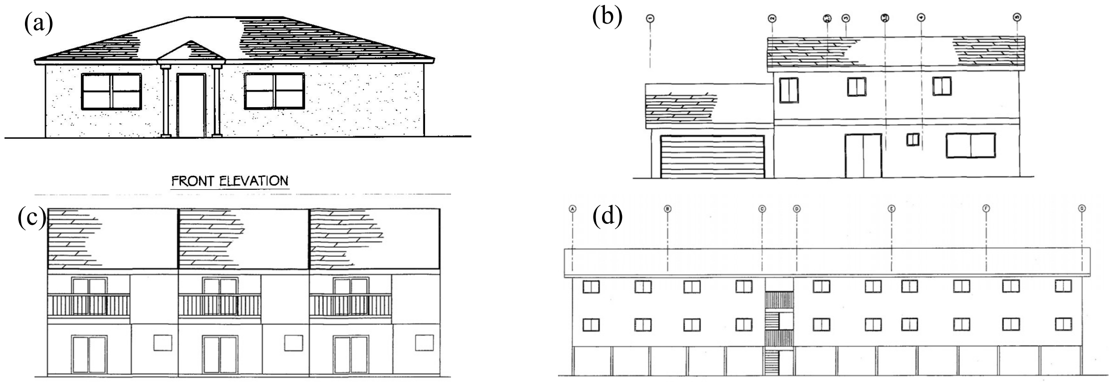

Figure 2 Index-building elevations (not to scale)

Quality-level variants. All four index buildings have three quality-level variants: poor, typical, and superior. Quality levels were defined by [Porter et al. 2002], and are quantified via structural member strength relative to laboratory conditions (typically 60%, 85%, and 100% for poor, typical and superior, respectively) and in terms of extra roof mass (3, 2, and 1 layer of built-up roofing in the poor, typical, and superior-quality variants, respectively). A professional construction cost estimator determined initial construction and retrofit costs (V and V_r of Equation 1) for all the variants, as well as repair costs for all assembly types and damage states ($C_{j,dm}$ of Equation 6), and the range of the overhead-and-profit factor C_{OP} (a and b of Equation 7).

Retrofit and redesign variants. In addition to quality-level variants, the small house has a retrofit variant: structural sheathing is added to the cripple walls, the sill plate is bolted to the foundation, and the water heater is strapped to the building frame. The large house has three alternative-design variants. In one, in the initial construction, structural sheathing is added to exterior walls above and below window and door openings (referred to as the "waist-wall" variant). In another, the initial design includes thicker, higher-grade sheathing and heavier, closer nailing to provide immediate-occupancy performance in a BSE-1 event as defined in FEMA 273 [FEMA 1997] (the "IO" variant). In the third, the initial design assumes rigid behavior of the 2nd-floor diaphragm ("rigid diaphragm"). The townhouse has a variant whose initial design has thicker sheathing to produce more-uniform interstory drifts ("limited drift"). The apartment building has two retrofit variants: one with steel moment frames added to garage openings ("steel frames"), another with structural sheathing added to the center longitudinal wall at ground floor (the "shearwall" variant).

Analysis

Ground motions. To reflect variability in ground motion, we drew on a set of horizontal-component pairs of 50 ground-motion time histories compiled for the SAC Steel project [Somerville et al. 1997]. The *IM* of Figure 1 is the 10%-damped elastic spectral acceleration at the building's estimated small-amplitude fundamental period T_1, calculated using a height-period relationship proposed by [Camelo et al. 2001]. We selected 20 ground-motion component pairs at random (without replacement) for each S_a from 0.1g, 0.2g, etc., up to 2.0g, scaling amplitudes to the desired S_a, subject to three constraints: we preferred records whose amplitudes could be scaled up or down by a factor of no more than 2.0 to match the desired S_a; we preferred domestic (U.S.) over foreign; and we preferred natural (recorded) ground motions over simulated ones. The scaling limitation was imposed on the advice of [Campbell 2001].

Structural analysis. Best-estimate structural models for each variant were prepared by [Isoda et al. 2001]. Shearwall force-deformation characteristics were calculated using the CASHEW finite-element software [Folz and Filiatrault 2001], and modeled as an equivalent nonlinear spring using a degrading-stiffness hysteresis model, including pinching [Stewart 1987]. (Hysteretic energy dissipation is included in the models in addition to viscous damping.) These springs were then used as elements in a whole-building structural model created for use in the structural-analysis package Ruaumoko [Carr 2001]. For each building variant, [Isoda et al. 2001] created a so-called pancake model, in which building diaphragms are represented as a flat deformable plates occupying the same plane, and shearwalls are represented by zero-height springs. We used these best-estimate models to create a stochastic structural model of each building, creating 20 simulations of each building, with mass and viscous damping varying randomly. Moments of the viscous damping ratio ($\mu_Z = 0.10$ and $\sigma_Z = 0.03$) were determined using data from forced-vibration tests of several real woodframe buildings by [Camelo et al. 2001]. Uncertain mass was modeled using moments offered by [Ellingwood et al. 1980]. Strength was modeled deterministically, via the quality characteristics. Each of 20 structural-model simulations was paired randomly (without replacement) with a ground-motion time-history at each of 20 levels of *IM*, and a structural analysis performed to calculate *EDP*.

Damage analysis. The *EDP*s of the structural analysis were input to fragility functions for each assembly to determine the cumulative probability distribution of *DM* for each assembly (Equation 4). We identified 13 distinct assembly types in the index buildings, each with up to four damage states. We developed fragility functions for each assembly type from various sources, notably tests of stucco walls [Chai et al. 2002]; woodframe walls [Pardoen et al. 2000]; gypsum wallboard partitions [McMullin and Merrick 2001]; and plywood and oriented strandboard (OSB) sheathing with stucco finish [Gatto and Uang 2001]. We also used theoretical glazing fragility [Sucuoglu and Vallabhan 1997]. The collapse probability of the small house and apartment building was modeled as function of the peak transient drift in the cripple wall (for the small house) or ground story (for the apartment building).

Loss analysis. Unit repair costs and overhead and profit were determined by a professional cost estimator, considering construction costs in ZIP Code zone 904 (within about 5% of the average for the state of California, per [RS Means 2001b]). We accounted for the cost to repaint rooms, hallways, and other lines of sight to reasonable uniform appearance, which generally can require repainting of undamaged surfaces. This expense, sometimes referred to as line-of-sight cost, sometimes referred to as reasonable uniform appearance, can be substantial, so it is important to consider it carefully. For collapse, it was assumed that the apartment building would be a complete loss if it collapsed, but that the small house could be jacked back into place and the necessary repairs performed for a cost of between approximately \$39,000 to \$51,000, per the cost estimator. The location cost factor C_L and inflation cost factor C_I are taken from [RS Means 2001b]. Resulting seismic vulnerability functions are shown in [Porter et al. 2002].

Validation. We compared the theoretical seismic vulnerability functions to historic earthquake experience data (Figure 3a). We used published data from three earthquakes: 1994 Northridge, 1983 Coalinga and 1971 San Fernando using [Steinbrugge and Algermissen 1990], [ATC 2001], [Schierle 2000], and [EQE and OES 1995], mapping from PGA to MMI in Figure 3a using [Wald et al. 1999]. We also compared with HAZUS (Figure 3b), using Table 5 from [Kircher et al. 1997]. Figure 3 shows generally good agreement for the small house. For other variants, see [Porter et al. 2002].

Seismic hazard. We calculated seismic hazard at every California ZIP-Code centroid, considering the area fraction of each site soil classification in each ZIP Code. We acquired USGS gridded hazard data [Frankel and Leyendecker 2001], which contain $G(s)$ at gridpoints of longitude (ϕ) and latitude (λ) spaced at $\Delta\phi = \Delta\lambda = 0.05°$ throughout California for s = 5%-damped elastic spectral acceleration at periods of T = 0, 0.1, 0.2, 0.3, 0.5, 1.0, and 2.0 sec, and BC NEHRP site soil classification. We interpolated $G(s)$ at ZIP-Code centroids [from GDT 2000a] by assuming the hazard varies between gridpoints according to a saddle shape. We adjusted for damping ratio by using Bispec [Hachem 2000] to analyze our ground-motion time histories [Somerville et al. 1997], and calculating the ratio of the 10%-damped to 5%-damped S_a. We calculated the area fraction of each California ZIP Code that has NEHRP site classification A, AB, B, etc., through E (recall p_{sc} of Equation 2), using site classifications from the California Geologic Survey [Wills et al. 2000], ZIP-Code boundaries taken from [GDT 2000b], and GIS software [ESRI Inc. 2001].

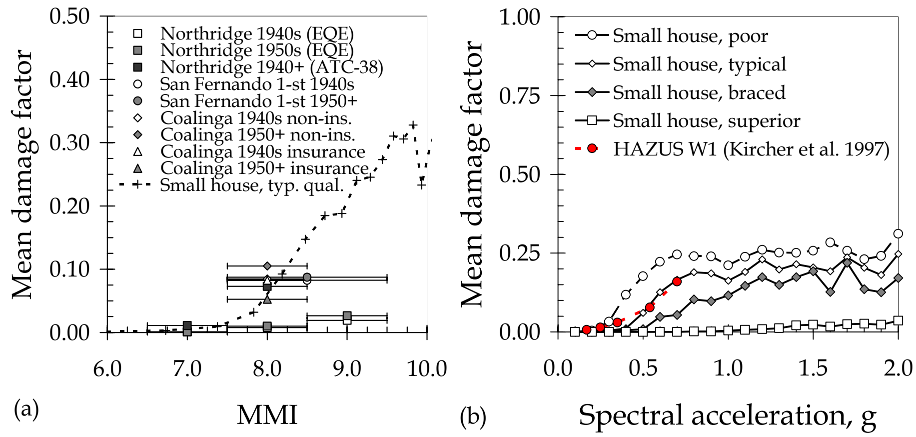

Figure 3 Small-house vulnerability functions versus (a) experience and (b) HAZUS

RESULTS

What if the owner of one of these buildings in a particular ZIP Code made one of the changes examined here, such as seismic retrofit of the small house, above-code design of the townhouse, or higher construction quality of the apartment building? What would be the economic benefit to the owner? (We do not consider here the benefit to society of a probabilistic mix of various buildings, but rather these particular buildings and these particular mitigation measures, in ZIP Codes throughout California.) We calculated the seismic vulnerability for each index building and variant, applied Equation 2 to determine expected annualized repair cost on a ZIP-Code basis, and calculated the benefit by Equation 1 for each retrofit and redesign measure, relative to the typical-quality variant.

Cost-effective retrofits and redesign measures. Four of the seven retrofit or redesign measures can be cost-effective in California. Figure 4 shows the calculated benefit and benefit-cost ratio (BCR) of the seismic retrofit of the small house: adding foundation bolts, structural sheathing to unbraced cripple walls, and strapping the water to the frame. The figure reflects a real (after-inflation) discount rate of 3%[1] and a planning period of 30 yr. The retrofit is cost-effective in half of California ZIP Codes. Figure 5a shows the benefit of above-code (limited-drift) design of the townhouse in the San Francisco Bay area. It is cost-effective in 300 ZIP Codes statewide, primarily in highly seismic coastal regions, with a present value of benefit as high as $8,000 per building. Both mitigation measures for the apartment building are estimated to be cost effective in various California locations; maps are omitted because of space constraints.

Benefit of construction quality. We find that construction quality has an important impact on earthquake losses. Some particulars: For the small house, the median benefit of typical-quality construction of the small house, relative with the poor-quality variant, is $3,000 (i.e., in 50% of ZIP Codes, the benefit exceeds $3,000). For the large house, the median benefit associated with superior quality (relative to poor) is $970. For the townhouse, the median benefit of superior-quality construction is $1,700, relative to poor. For the apartment building, the figure is $13,000; in one ZIP Code, the benefit is $120,000. Figure 5b shows the benefit of superior versus poor construction quality for the apartment building in the Los Angeles area.

Losses other than repair costs are ignored. The BCRs shown here are based solely on repair costs. If we accounted for content loss, loss of use, human injuries, etc., BCRs could be much higher. Including these

[1] 6% mortgage interest rate [California Department of Finance 2004] less 3% inflation [BLS 2004].

additional benefits would be a simple extension of the methodology presented here, assuming that the required fragility and other data are available.

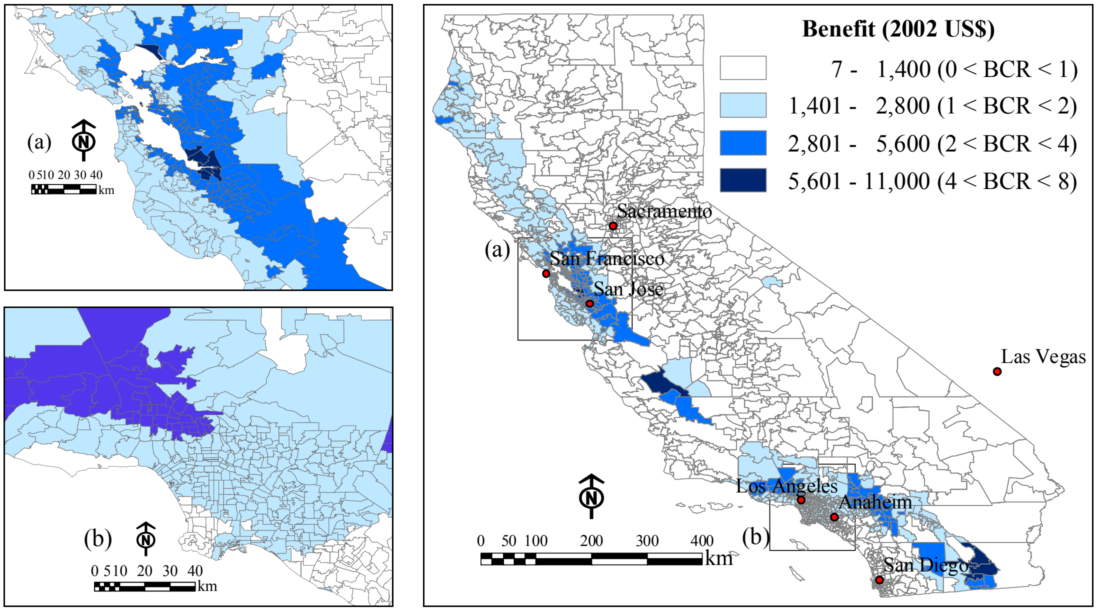

Figure 4 Benefit of small-house retrofit

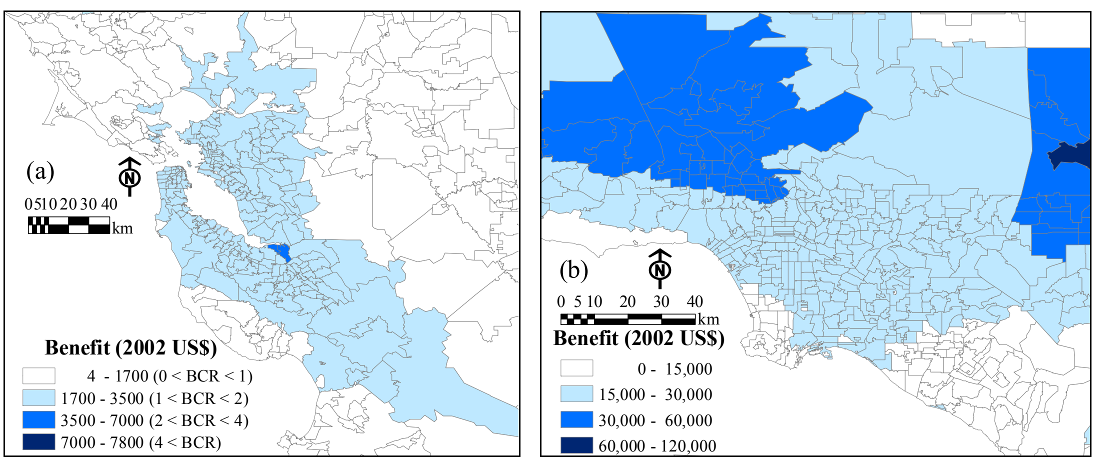

Figure 5 (a) Benefit of above-code (limited-drift) design of townhouse and (b) of superior-quality construction of the apartment building, relative to poor quality

CONCLUSIONS

We developed probabilistic seismic vulnerability functions for 19 fully-designed woodframe dwellings. We employed laboratory tests and analytical tools developed under the CUREE-Caltech Woodframe Project; rigorously propagated important sources of uncertainty; used nonlinear time-history structural analyses; clearly accounted for repair costs including line of sight; and avoided reliance on expert opinion. Each vulnerability function includes information on the repair-cost distribution conditioned on S_a and reflects 400 independent simulations of structural response, damage, and repair cost. Using soil maps produced by the California Geologic Survey and hazard data by the US Geological Survey, we calculated expected annualized loss for each building, calculated the benefit-cost ratio for seven detailed retrofit and redesign measures, and the benefit of higher construction quality, all on a ZIP-Code basis.

Four of the retrofit and redesign measures are estimated to be cost effective in various locations throughout California—generally near faults and on soft soil, as expected. When examining the benefit of higher-quality construction, we found that the savings in terms of reduced seismic risk can be substantial, with median savings on the order of $1,000 to $10,000 over 30 yr, suggesting a quantitative argument for frequent construction inspection. These results ignore benefits such as reduced content damage, reduced loss of use, and human injuries avoided. Were these benefits included, benefit-cost ratios could be substantially greater. These benefits are easily included in the methodology, given the appropriate data.

The data presented here can be used to inform risk-management decisions by homeowners, engineers, and public officials. Homeowners can use this information to decide if retrofit is likely to be worth the expense. Engineers can use the data in the development of code requirements. Public officials can use the data to target particular dwelling types and geographic locations for public-awareness programs that promote retrofit where it is likely to be cost effective. We have not tested how well the maps such as those shown in Figure 4 and Figure 5 convey useful risk information to these decision makers; that would be valuable research.

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